Fermilab

# Particle Physics Division
# Mechanical Department Engineering Note

Number: MD-ENG-141 Date: 22 January 2008

Project Internal Reference: none

| | |
|---|---|
| Project: | Minerva Test Beam Detector |
| Title: | Minerva Test Beam Detector Structural Steel Support |
| Author(s): | Dave Pushka |
| Reviewer(s): | |
| Key Words: | Structural Steel |
| Abstract Summary: | Structural Steel calculations for the test beam detector support. |
| Applicable Codes: | AISC $9^{th}$ Edition, ASD |

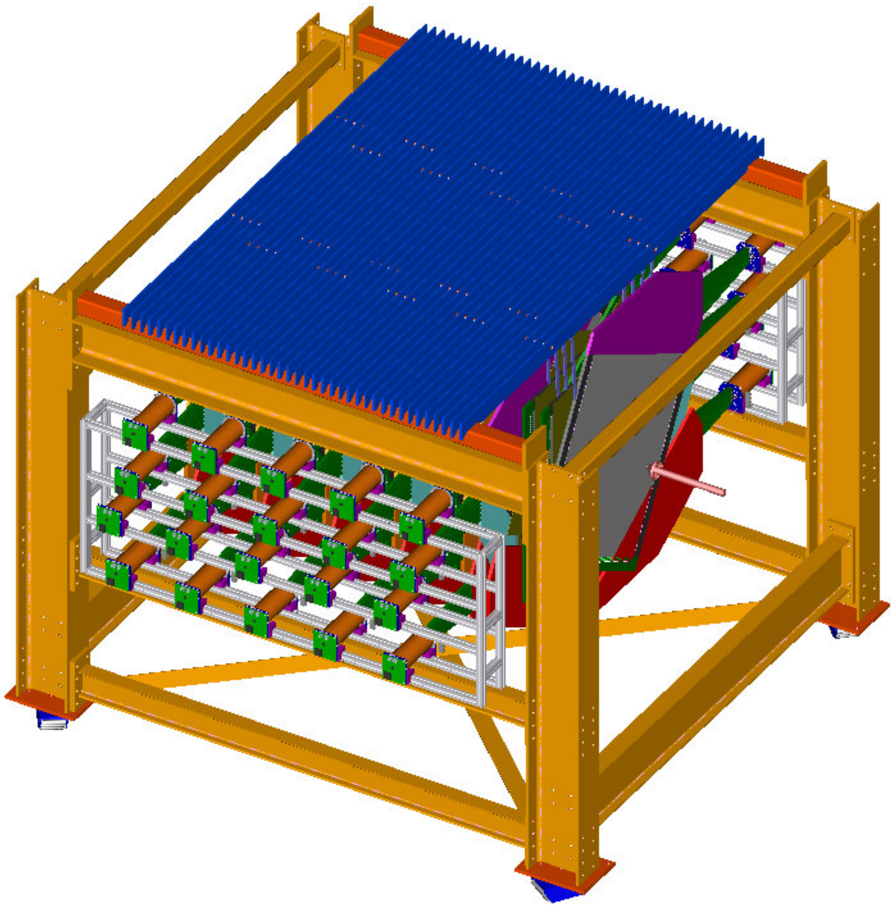


Image 1: Minerva Test Beam Detector in Structural Steel Support Frame.

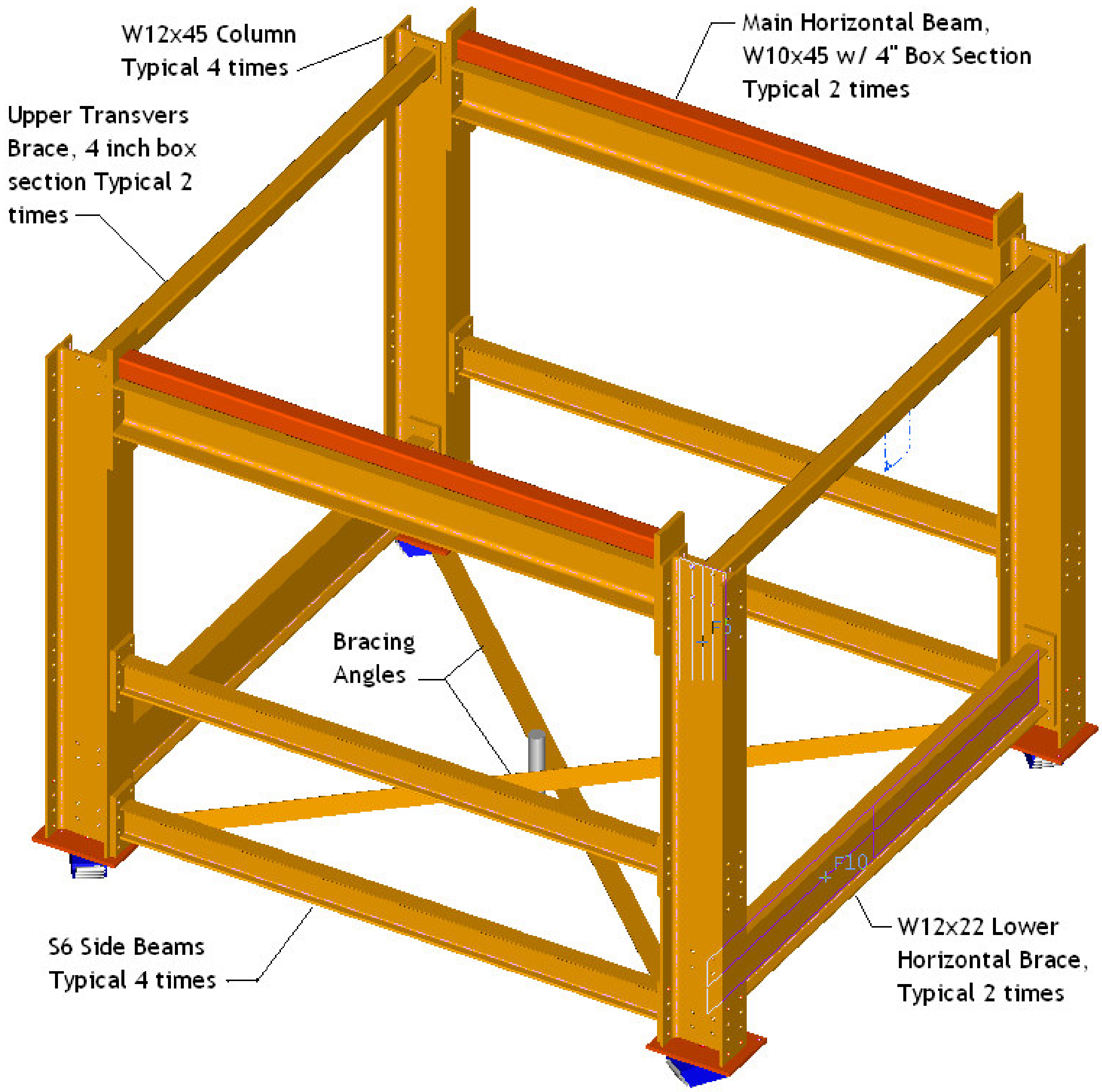



Image 2: Minerva Test Beam Detector Support Frame

**Columns:**

Four W12 by 45#/ft sections are re-used for the columns. Since the pedigree is unknown, assume A36 material. Assume rotations are allowed at both column ends, therefore, k = 1.0

Column length is 8 feet. Therefore, KL = 1,0 * 8 = 8. From AISC 9th edition, page 3-28, the allowable axial load for each column is 243 kips. A = 13.2 square inches. Therefore, the allowable axial stress, Fa = 18.4 ksi. Actual load on the column is 7 kips. Axial stress on column, $f_a$ is 7 kips over 13.2 inches = 530 psi.

Columns are not loaded axially; rather the main horizontal beams apply the load to one flange face. Resulting eccentricity is 6 inches. Using a 7 kip eccentric load (see the sizing of the main horizontal beam for the source of this load), the moment is 42 kip-in. From AISC 9th Edition, chapter H on page 5-54, when $f_a / F_a$ is < 0.15, Equation H1-3 may be used. $f_a / F_a + f_{bx} / F_{bx} + f_{by} / F_{by} < 1.0$.

Actual Bending stress, $f_{bx}$ = My/I = 42 kip-in*6 inches / 350 in^4 = 0.72 ksi.
Allowable Bending stress, $F_{bx} = 0.6*F_y$ = 21.6 ksi
Actual Bending stress, $f_{by} = 0$
$f_a / F_a + f_{bx} / F_{bx} + f_{by} / F_{by}$ = .53 / 18 + .72 /21.6 + 0 = 0.0628 << .15 Therefore, okay.

Conclusion on columns: Columns have a capacity that greatly exceeds the load because of the choice to re-use existing, large columns.

**Main horizontal Beams:**

Two W10 by 45#/ft sections are re-used for the main horizontal beams. Since the pedigree is unknown, assume A36 material. Span, l, is 106 inches. Loading condition is described by load case 4 on AISC 9th edition, page 2-297 (Assumes free ends). From work presented by Robert Flight, the load on the structure uses about 78 inches of the span length and places 28 kips, evenly distributed between each of the two main horizontal beams.

Ignore contribution from 4 inch box section. Use I = 248 in^4 for the W10x 45

l = 106 inches
b = 78 inches
a = c = (106-78)/2 = 14 inches
w = 28/2 kips /78 inches = 179 pounds per inch
R1 = R2 = 7 kips
Maximum bending moment, $M_{max}$ = R1*(a+R1/2w) = 7 kips *(14+7/2*.179) = 234.5 kip-in
Bending Stress, Sigma, = My/I. = 234.5 kip-in*5/248 in^4 = 4.72 ksi.

If the ends are considered to be restrained against rotation (see case 15 on page 2-301) the bending moment becomes:

$M_{max}$ = w * l^2 /24 = 179 pounds per inch, * (106 inches)^2 / 24 = 83,801 in-lbs
Bending Stress, Sigma, = My/I. = 83801 kip-in*5/248 in^4 = 1.69 ksi.

Use the simple end results as this gives the highest stress.

Shear Stress, $\tau = R_1/A$ = 7 kips / 13.3 in^2 = 0.526 ksi.

| | |
|---|---|
| Allowable bending stress is 0.6 $F_y$ = 0.6 * 36 ksi = | 21.6 ksi |
| Actual bending stress is: | 4.7 ksi |

Allowable bending stress exceeds actual bending stress, therefore, acceptable.

| | |
|---|---|
| Allowable shear stress in beam is 0.4 * $F_y$ = 0.4 * 36 ksi = | 14.4 ksi |
| Actual shear stress in beam is: | 0.526 ksi |

Allowable shear stress exceeds actual shear stress, therefore, acceptable.

Use the formula for a uniformly loaded beam to estimate the deflection. For a 78 inch long, uniformly loaded member; Maximum Deflection, $\delta_{max}$= 5/384 wl^4/EI
$\delta_{max}$= (5/384) * (0.179 kip/in * 78in^4) / 29 x 10^3 ksi * 248 in^4 = 0.012 inches

**Connection between Columns and Main Horizontal Beams:**

Use a standard bolted end plate shear connection. See AISC 9th edition, pages 4-49 thru 4-51.

Assume A307 bolts are used (equivalent to grade 2 bolting) as this avoids the reliance on high strength bolting and the susceptibility to suspect or counterfeit (S/C) material.

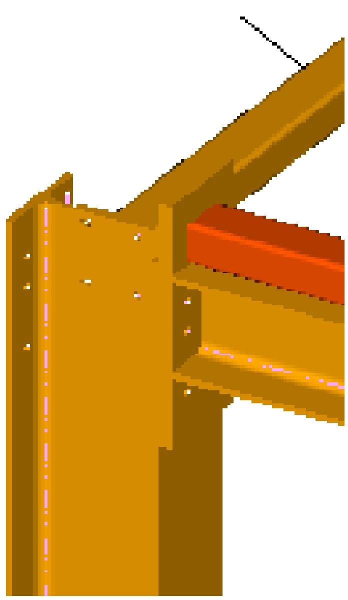
6 bolts in single shear carry the load.

Use ¾ inch nominal diameter bolts.
Each connection has a load of 7 kips.
Each connection has six bolts sharing the 7 kip load in single shear.
Each bolt sees 1167 pounds in single shear.
Allowable load per bolt from AISC TABLE I-D is 4400 pounds.
Allowable load for the bolts used in the connection is 26.4 kips

Use 1 inch thick plate as 1 inch thick material is available. This exceeds the thickness required on page 4-51.

Weld the angle to the main horizontal beam using a ¼ inch weld size, 8½ inch long angle, the allowable capacity is 49.5 kips based on a minimum beam web thickness of 0.51 inches. Actual beam web thickness is 0.350 inches. Therefore, the actual allowable load is (0.35/0.51)* 49.5 kips = 33.9 kips.

| | |
|---|---|
| Actual load on connection: | 7 kips |
| Minimum connection allowable capacity: | 17.6 kips |



Connection allowable capacity exceeds the actual load, therefore, acceptable.

**Lateral Load Capability:**

Assembled detector will be moved when fully loaded. Movement will be quasistatic. Therefore, the lateral load capability will be dictated by the seismic criteria. In north east Illinois, a lateral load capability equal to 6% of the structure weight is sufficient to meet seismic criteria.

Assume detector weight is 28 kips and the structure adds another 7 kips. Use a total weight of 35 kips. 6% of 35 kips is 2.1 kips.

First, consider the lateral load in the direction perpendicular to the normal beam direction (transverse) and apply the 2.1 kip lateral load equally to both the front two columns (and W12x22) section connecting them.

Elevation of lateral load is assumed to be at the center of gravity of the assembly and the center of gravity is assumed to at the beam elevation of 61 inches above the column base plates.

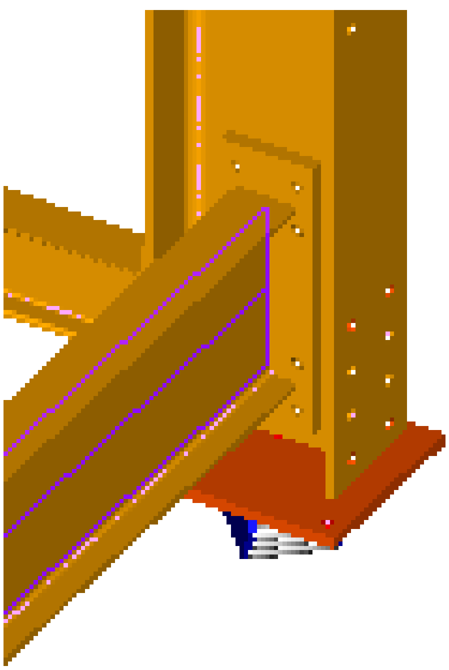
Moment applied is 61 inches * 2.1/2 kips is 64050 in pounds. For a W12 x 22, $I_{xx}$ is 156 in4. Bending stress, $\sigma$ = My/I = 64050 in-pounds*6 inches / 156 in^4 = 2463 psi.

Allowable bending stress is 0.6 * $F_y$ = 0.6 * 36 ksi = 21.6 ksi. 2463<<21,600 psi, therefore, the lower (transverse) horizontal brace is satisfactory to provide the restraining moment needed to counter the lateral load in the direction perpendicular to the beam.

**Check Moment connection of W12x22 to Column W12*45**:
Refer to AISC 9th edition, page 4-116 for an End Plate Moment connection. Use a 4-tension bolt design. $P_f$ = 1.5 inches. Use $b_p = b_{fb} + 1$ = 5 inches (actual width is 6 inches). Applied moment is 64.05 kip-in. Nominal depth is the depth of the W12x 22 and is 12 inches. Flange force, $F_f$ = M/nominal depth = 64.05 kip –in / 12 in = 5.33 kips. With four tensile bolts, bolt force is 5.33 kips / 4 = 1.33 kips per bolt. Therefore, 5/8 inch bolts are sufficient as A307 material has an allowable tensile stress of 6.1 kips for this size.

End Plate thickness, $t_p$ = SQRT((6 * $M_e$)/(0.75$F_y$*$b_p$))
End Plate Moment, $M_e$ = Bolt Tension * lever arm / 2
$M_e$ = 1.33 kips * 2 inches /2 = 1.33 kip-in
$t_p$ = SQRT((6 * 1.33 kip-in)/(0.75*36*6)) = 0.22 inches

Actual plate thickness will be 1 inch because this material is available.
Actual bolting will be ¾ inch A307 because these bolts are available.



Detail of a moment connection between the bottom of the column and the W12 x 22 transverse brace. Plate is 1 inch thick.

A total of eight ¾ inch A307 bolts are used at each connection. Bolt centerlines straddle the upper and lower flanges of the W12 x 22.

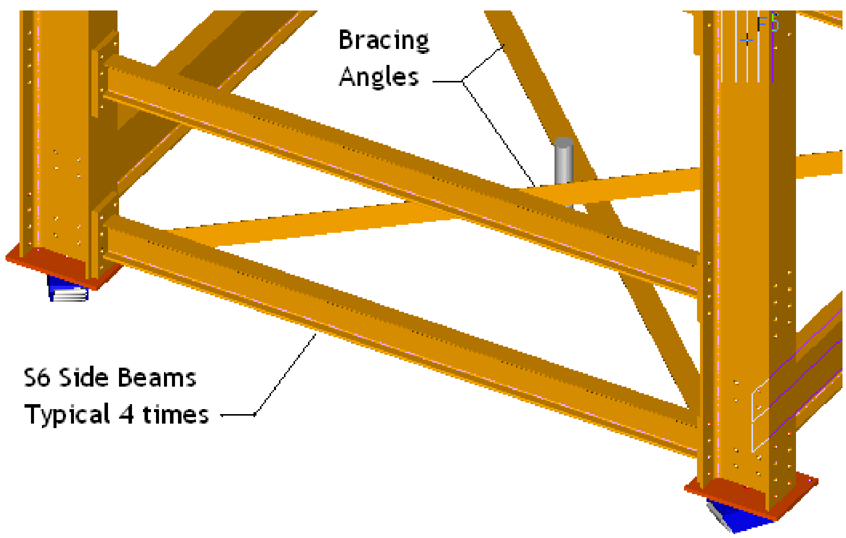
Second, consider the lateral load in the direction parallel to the normal beam direction and apply the 2.1 kip lateral load equally to both the front two columns (and S6) section connecting them.

Elevation of lateral load is assumed to be at the center of gravity of the assembly and the center of gravity is assumed to at the beam elevation of 61 inches above the column base plates.

Moment applied is 61 inches * 2.1/2 kips is 64050 in pounds.

For a S6 x 17.5, $I_{xx}$ is 26.3 in4. Bending stress, $\sigma$ = My/I = 64050 in-pounds*3 inches / 26.3 in^4 = 7306 psi.

Allowable bending stress is 0.6 * $F_y$ = 0.6 * 36 ksi = 21.6 ksi. 7306 < 21,600 psi, therefore, the lower horizontal brace (side beams) is satisfactory to provide the restraining moment needed to counter the lateral load in the direction perpendicular to the beam.



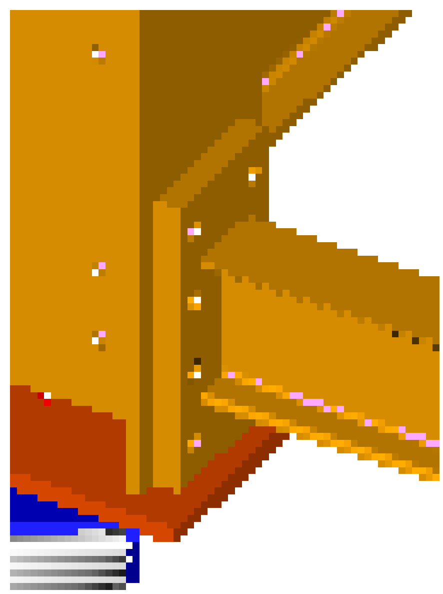

Note that 2 S6 x 17.5 side beams are used on each side of the detector. While one is needed to transfer moment for lateral stability, the other (upper) is needed to support the readout electronics. For simplicity, the two beams are detail with moment connections and together provide lateral stability.

**Check Moment connection of S6 x 17.5 to Column W12*45**:

Refer to AISC 9th edition, page 4-116 for an End Plate Moment connection. Use a 4-tension bolt design. $P_f$ = 1.5 inches. Use $b_p = b_{fb} + 1$ = 5 inches (actual width is 6 inches). Applied moment is 64.05 kip-in. Nominal depth is the depth of the S6x 17.5 is 6 inches. Flange force, $F_f$ = M/nominal depth = 64.05 kip –in / 6 in = 10.67 kips. With four tensile bolts, bolt force is 10.67 kips / 4 = 2.66 kips per bolt. Therefore, 5/8 inch bolts are sufficient as A307 material has an allowable tensile stress of 6.1 kips for this size.

End Plate thickness, $t_p$ = SQRT((6 * $M_e$)/(0.75$F_y$*$b_p$))
End Plate Moment, $M_e$ = Bolt Tension * lever arm / 2
$M_e$ = 2.66 kips * 2 inches /2 = 2.66 kip-in
$t_p$ = SQRT((6 * 2.66 kip-in)/(0.75*36*6)) = 0.44 inches

Actual plate thickness will be 1 inch because this material is available.
Actual bolting will be ¾ inch A307 because these bolts are available.



**Moving Loads:**

When the assembly is moved, the rollers will resist movement due to friction. It is anticipated that the force needed to move the assembly will be provided by a Johnson bar, hydraulic cylinder, or other means applied to the bottom of one of the four main vertical columns.

Friction between the bottom of the roller and the floor bearing surface is about 5% just before motion occurs. As soon as the rollers begin to move, the static friction drops significantly. This leads to the familiar stick-slip motion that very slow moving, heavy loads exhibit.

Assume that a friction force of 5% of 28 kips of detector weight and 5% of the support structure weight.

Support structure weight:

| Member | Weight per ft | Length, ft | Number | Gross weight |
|---|---|---|---|---|
| column | 45 | 8 | 4 | 1440 |
| Main Side beam | 45+21.63 | 9 | 2 | 1200 |
| Lower Transverse beam | 22 | 9 | 2 | 400 |
| S6 Side Beam | 17.5 | 9 | 4 | 630 |
| Bottoms Angle brace | 12.8 | 12 | 2 | 308 |
| TOTAL | | | | 3978 pounds |

5 % of the support structure weight is about 200 pounds
5% of the detector weight is 1400 pounds

Sum is 1600 pounds.

Assume each lower brace angle can only see tension (ignore any compressive strength) and area of one angle is 1.94 inches ( this area is for a angle 4 x 4 x ¼ size while the above weight is for a 4 x 4 x ½ size). Allowable tensile load is 0.4 * $F_y$ = 0.4 * 36 ksi = 14,400 psi. Actual tensile stress is 1600 pounds / 1.94 inches = 824 psi. Since the actual load is much less than the allowable load, this bracing is more than sufficiently sized.